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Design of a Steel Head-Frame

Civil Engineering

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DESIGN

OF A

STEEL HEAD-FRAME

BY

DJALMA DOWNEY WILLIAMS

THESIS

FOR

DEGREE OF BACHELOR OF SCIENCE

IN

CIVIL ENGINEERING

COLLEGE OF ENGINEERING

UNIVERSITY OF ILLINOIS

PRESENTED JUNE, 1907

C O L L E G E O F E N G I N E E R I N G

April 30, 1907.

This is to certify that the following thesis prepared under the immediate direction of Professor F. O. Dufour, Assistant Professor of Structural Engineering, by

DJALMA DOWNEY WILLIAMS

entitled DESIGN OF A STEEL HEAD-FRAME FOR A MINE

is accepted by me as fulfilling this part of the requirements for the Degree of Bachelor of Science in Civil Engineering.

Ira O. Baker

Head of Department of Civil Engineering.

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TABLE OF CONTENTS.

ART. I. INTRODUCTION.

ART. 2. SPECIFICATION.

ART. 3. DETERMINATION OF STRESSES.

Wind Stresses.
Live Load Stresses.
Dead Load Stresses.

ART. 4. PRELIMINARY DESIGN.

ART. 5. FINAL DESIGN.

Main Posts.
Lateral Bracing.
Sheaves and Axle.

ART. 6. ESTIMATE.

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ART. I. INTRODUCTION.

The subject of this thesis, "The Design of A Steel Head Frame" is one of the new problems that have been presented to structural engineers within the last few years.

It has been but a very short time since all headframes were constructed entirely of wood. At that time there was reason for this, such, as abundance of suitable timber, ease of construction, and the reluctancy of mine owners to depart from the practice of the day.

However with the growing scarcity of timber, and with the

great advances made in engineering, it was seen that steel, for this kind of construction had many advantages over timber, and so it was accordingly adopted.

The purpose of the writer is not to make a design for any particular mine, but to make a design for a mine having a daily output of from 400 to 600 tons, the depth being about 500 feet.

ART, 3. SPECIFICATION.

This steel-head frame is to be designed according to Theodore Cooper's General Specifications for Steel Highway and Electric Railway Bridges and Viaducts, revised edition of 1901; with the following modifications.

General.

The head-frame is to be designed for a shaft 500 ft deep having double compartments, each compartment being 4 ft × 4 ft in the clear, with 12 inch timber walls, and partition.

Dimensions.

The height of the head-frame is

to be 60 ft from top of foundations to the centre of the sheaves. The base shall be 30 ft by 22 ft measured upon the foundations from the centre of the main posts.

Cable.

A 1¼ inch crucible steel cable having an ultimate stress of 116 000 #, and with an allowable working stress of 24 000 lb, is to be used.

Weight of carriage, car and cable.

Weight of car is 2000 lbs when empty and 6000 lbs when loaded. Weight of hoisting carriage is 4000 lb and weight of the cable is 2000 lb.

It is obvious that the maximum weight of 12 000 lbs comes on the cable

when a loaded car is starting from the bottom of the shaft. allowing a factor of safety of two to account for stresses due to changes in acceleration, as for instance these stresses due to starting, stopping, and nedging, this gives an allowable working stress of 24000 lb in the hanger with the allowable working stress of 24000 lb for a 1¼ inch cable as given above.

Sheaves.

There are to be two sheaves, each nine feet in diameter.

The sheaves are to have cast iron rims and wrought iron spokes, and each is to be designed as to safely bear the breaking stress of the cable.

Dead Load.

The dead load is to be taken as the dead weight of the structure.

Live Load.

The live load is to be taken as a breaking stress on one cable and a maximum working stress with a factor of safety 2 upon the other cable, i.e. 116000 lb + 24000 lb = 140000. lb on 10 sq in.

Wind Load.

The wind load is to be taken at 30 lb per sq ft of vertical projection, and is to be considered on all four sides in succession.

The wind bracing shall be so designed as to take both tension, and compression and all connections are to be riveted.

In all stresses the plus sign (+) will be used to denote tension, and the minus sign (−) to denote compression.

Fig. 2.

f=0.8 = 11.200 lbs

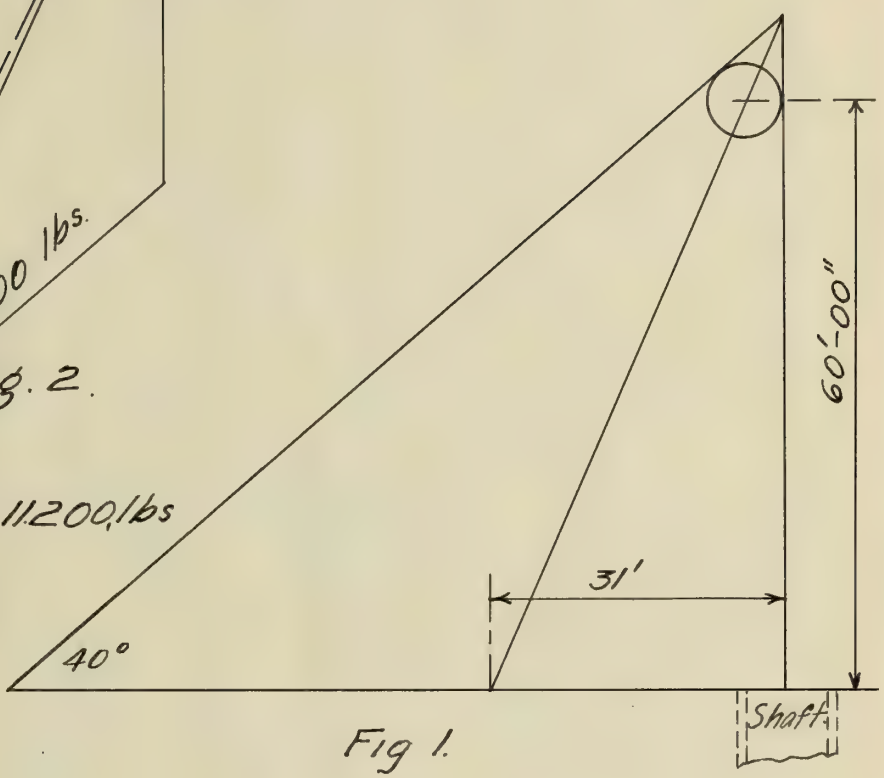
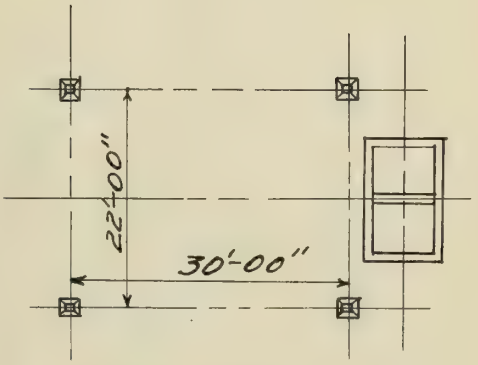


Fig 1.



Ground Plan.

Fig 3.

DETERMINATION
of
RESULTANT
under
Maximum Loading.

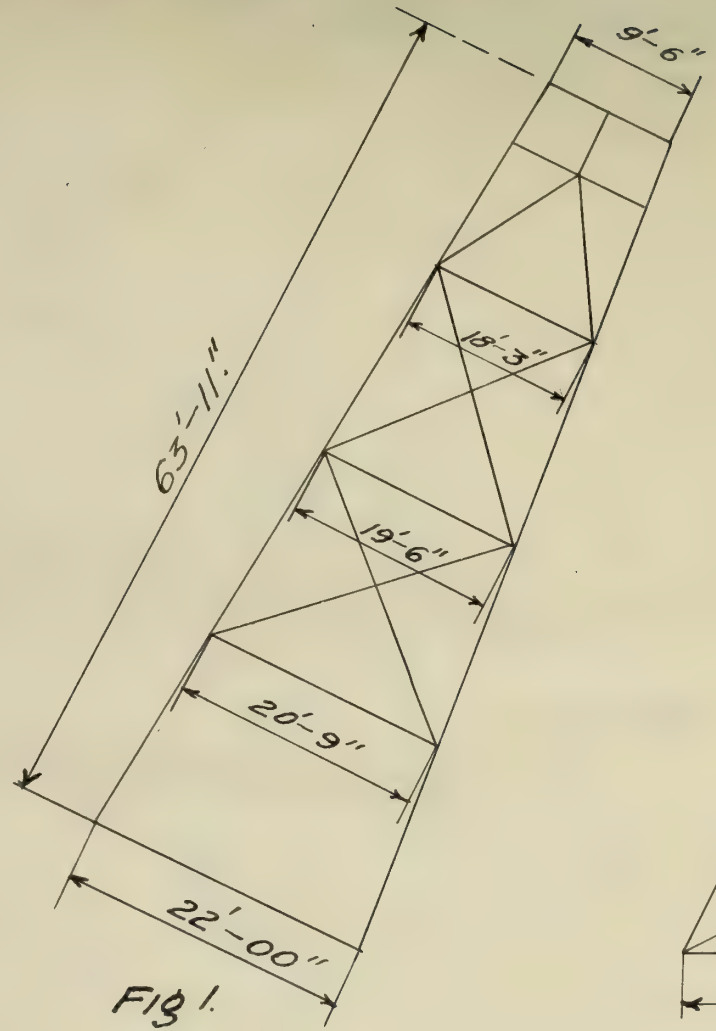


Fig 1.

Scale 0 5 10 15 20

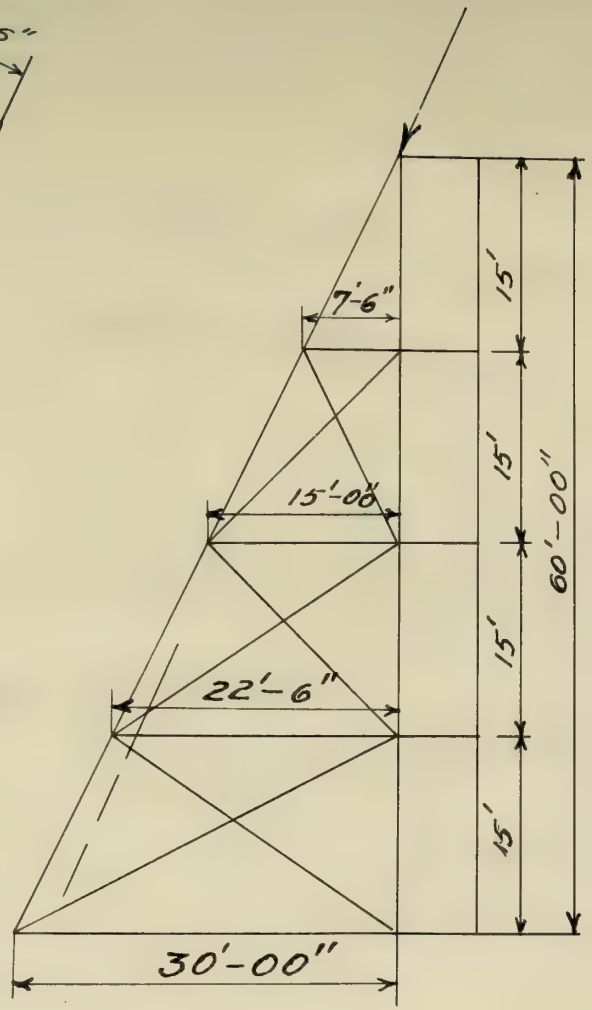


Fig. 2.

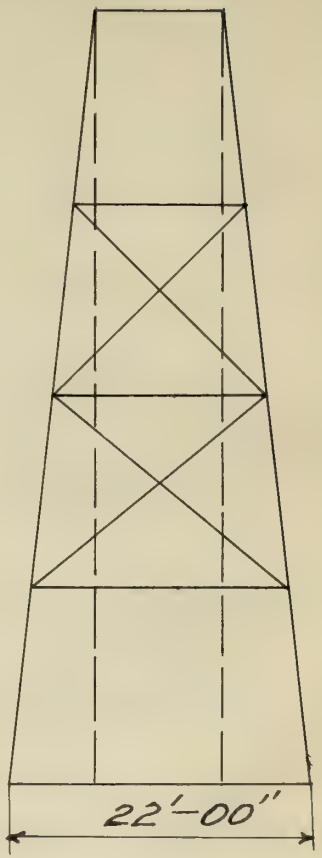


Fig. 3.

OUTLINE DIAGRAM.

PLATE III.

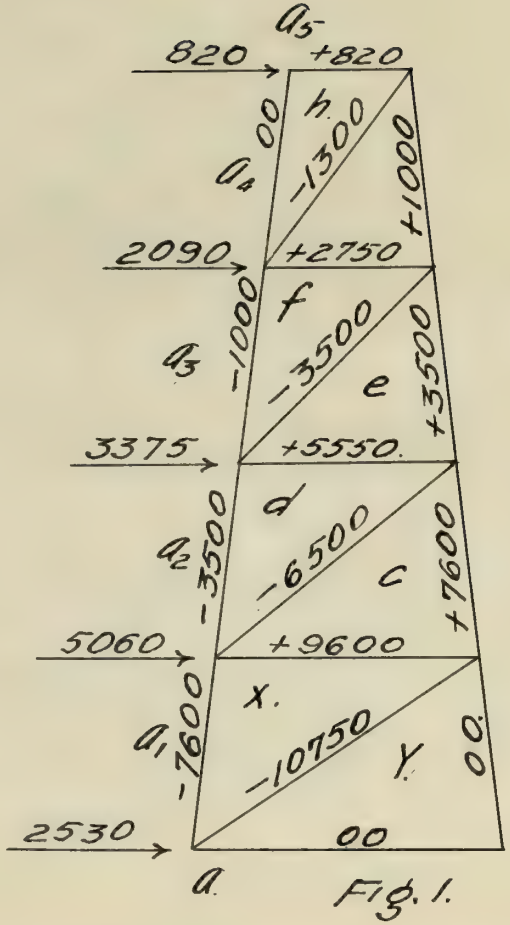


Fig. 1.

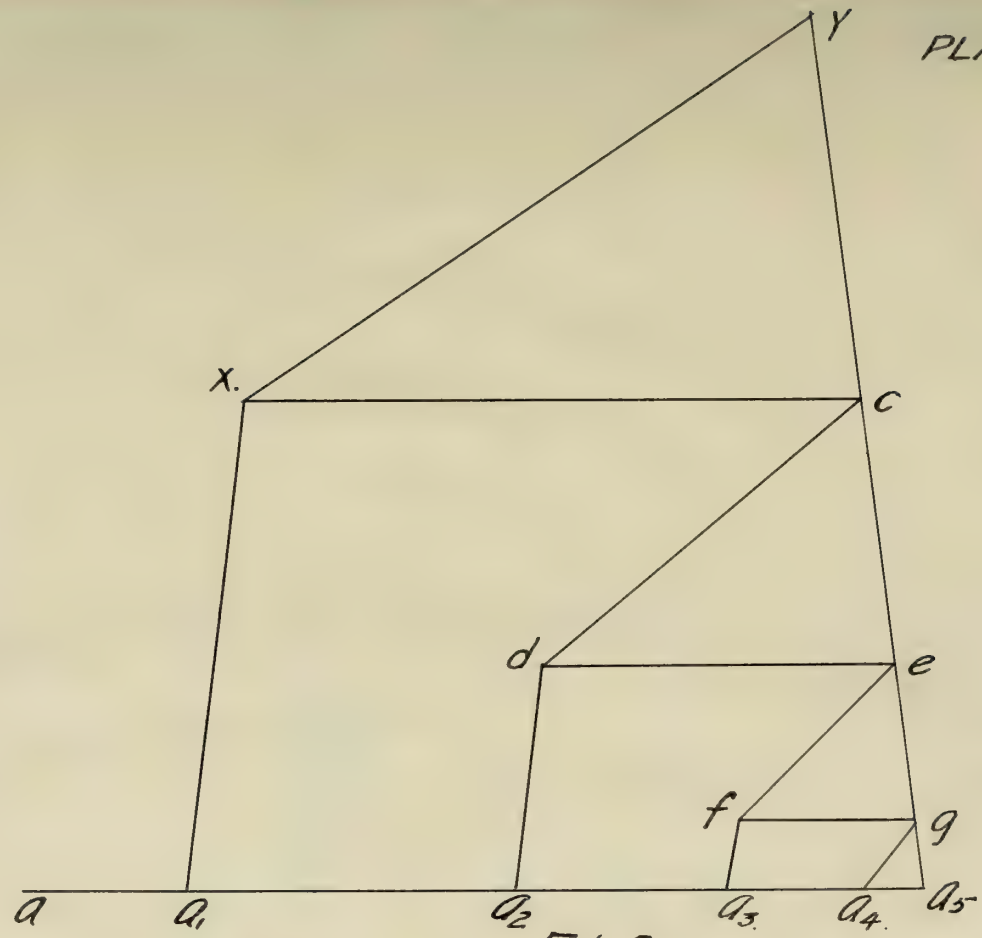


Fig-2:

STRESS DIAGRAM.
for
WIND LOAD
ON FRONT BENT.

Scale 0. 1. 2. 3. 1000 lbs.

PLATE. IV.

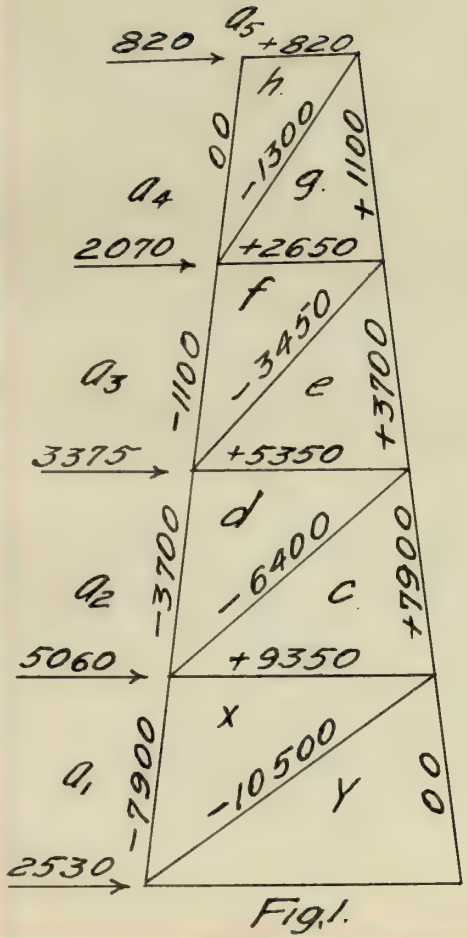


Fig. 1.

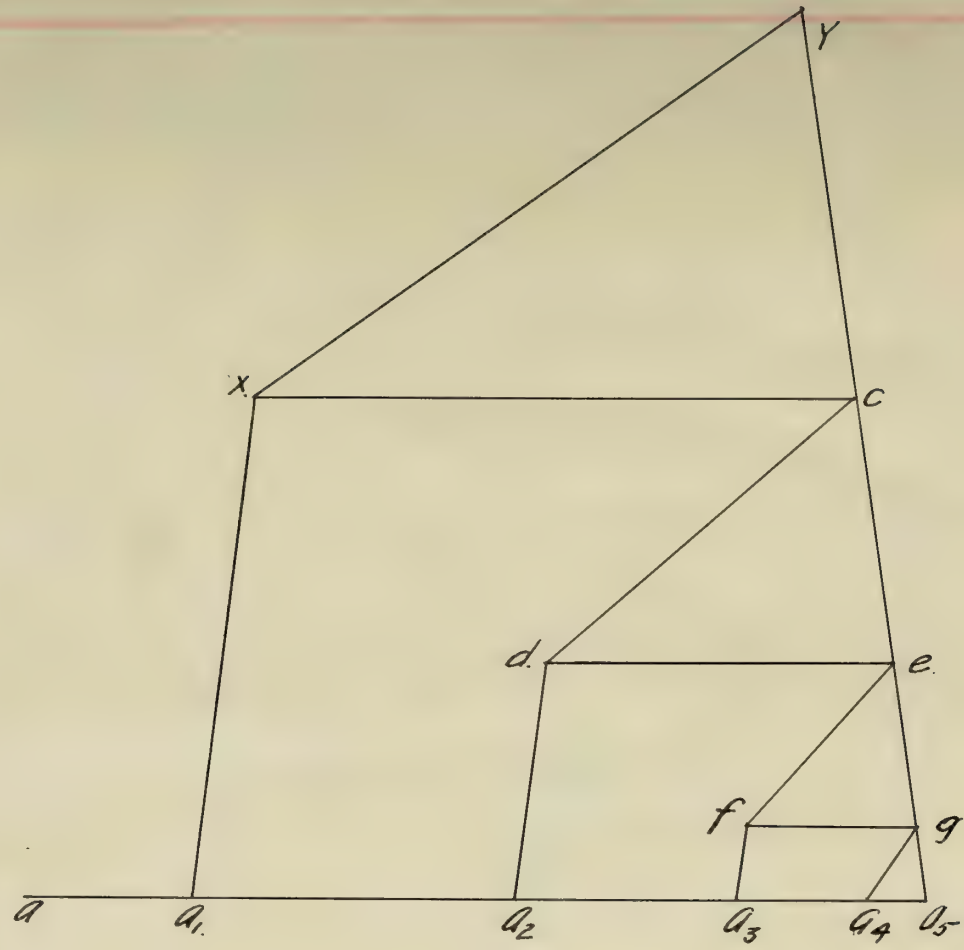
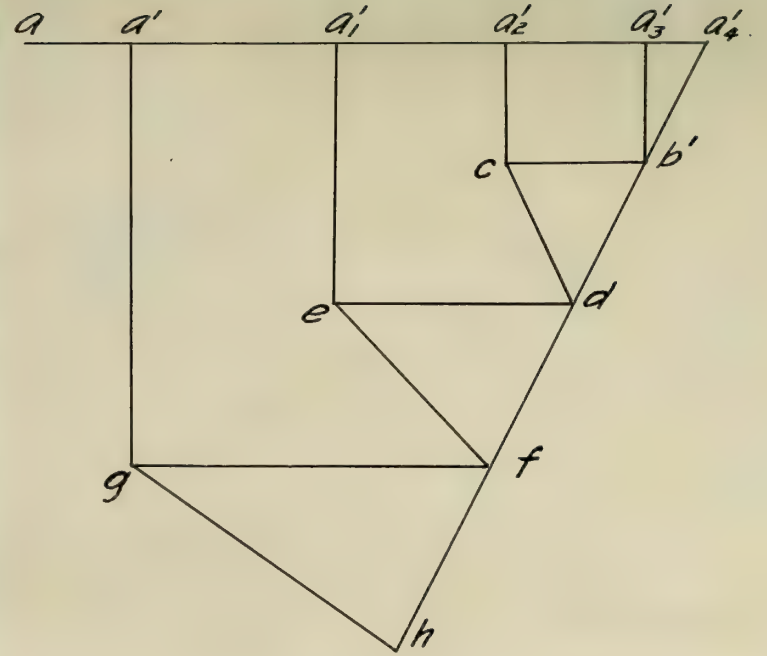


Fig. 2.

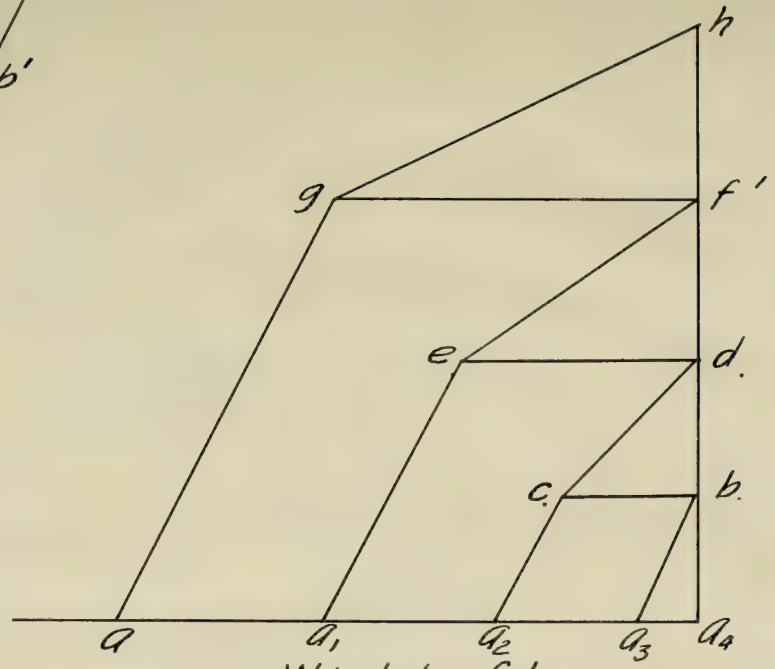
STRESS DIAGRAM.
for
WIND LOAD
ON BACK BENT.

Scale 0. 1. 2. 3. 1000 lbs.

11

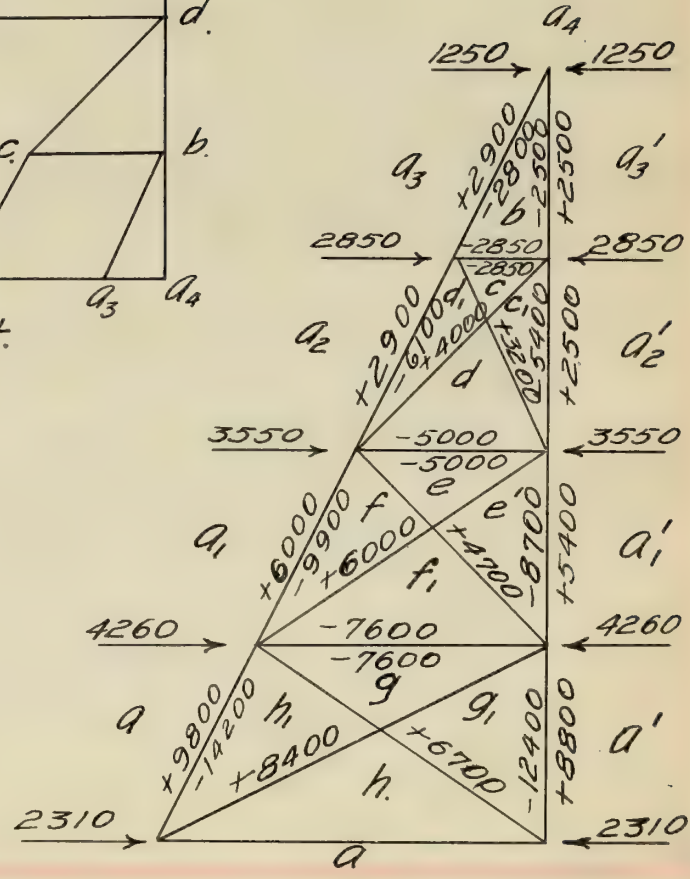


Wind Right.



Wind Left.

STRESS DIAGRAM
for
WIND LOAD
0 1 2 3 4 1000 lbs.



12.

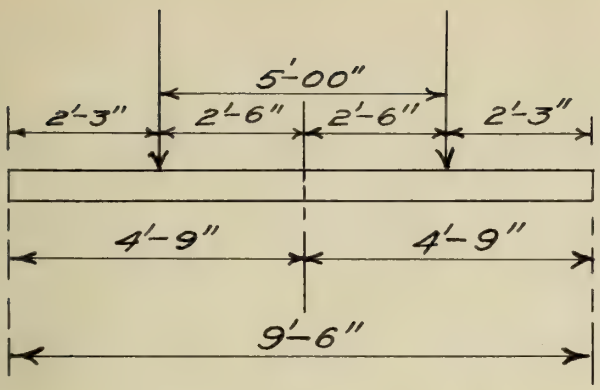
ART. 3. DETERMINATION OF STRESSES.

A.- Wind Stress.

A wind force of 30 lbs per sq ft is considered acting in succession on the right and left of the headframe, as shown in Fig 3 Plate II.

For the first, or bottom panel, there is a wind force of 11800 lbs; for the second panel the wind force is 8400 lbs; for the third panel the wind force is 5000 lbs. In each of these three panels the load is considered as being equally divided among the four panel points. In the fourth or top panel the wind force is 1840 lbs, one half of this is applied at the two lower panel points and the remaining half is applied at the top of the panel. The determination of the wind stresses in the various

PLATE. V. a.



Axle of Sheave.
Fig. 1.

M_e $4.75 \times S_2 - 24 \times 2\frac{1}{2} = 0$

$S_2 = 12.6$

M_d $9.5 \times 12.6 + S_1 \times 4.75 - 116 \times 2\frac{1}{4}$
$-24 \times 7\frac{1}{4} = 0$

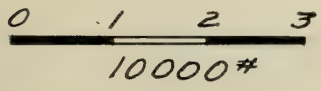
$S_1 = 66.0$

$S + S_1 + S_2 - 116 - 24 = 0$

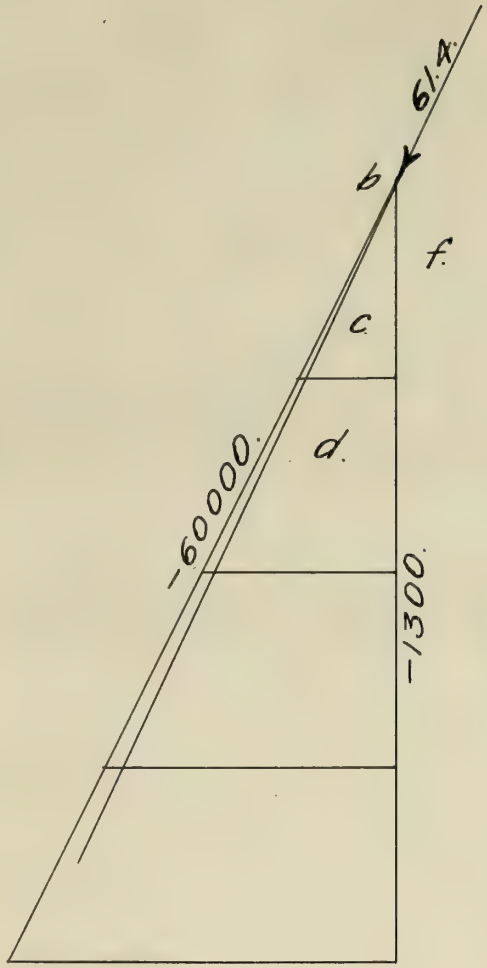
$S = 61.4.$

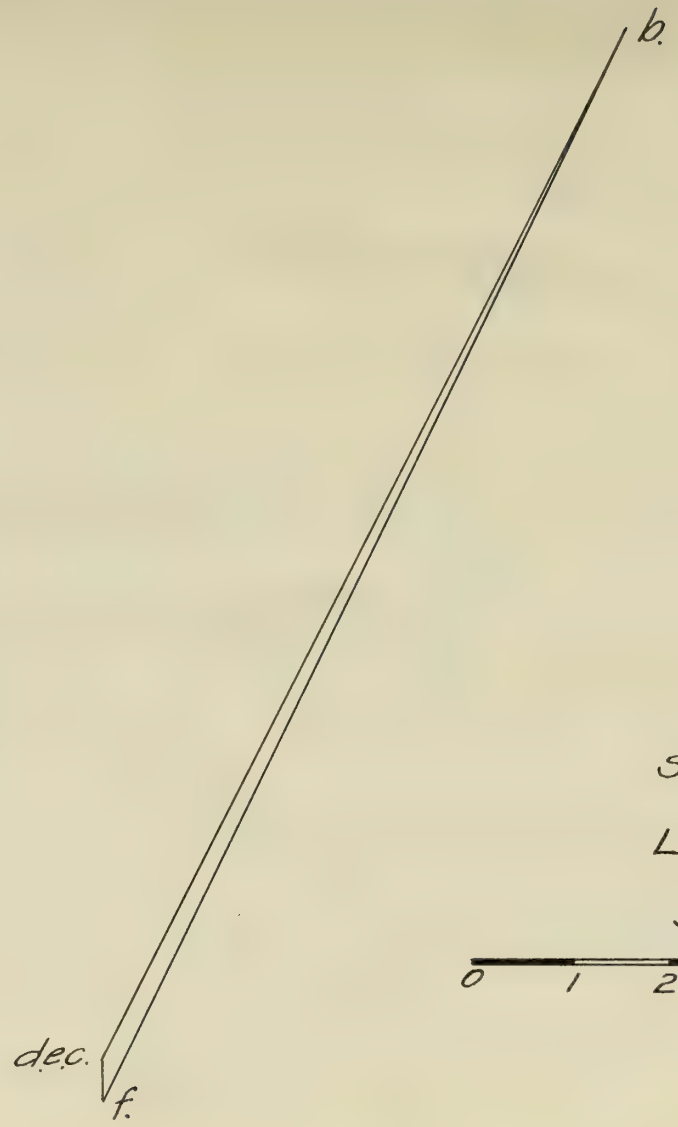
Fig 2.

Horizontal thrust is
$= 6300 - 1300 = 5000^{\#}$



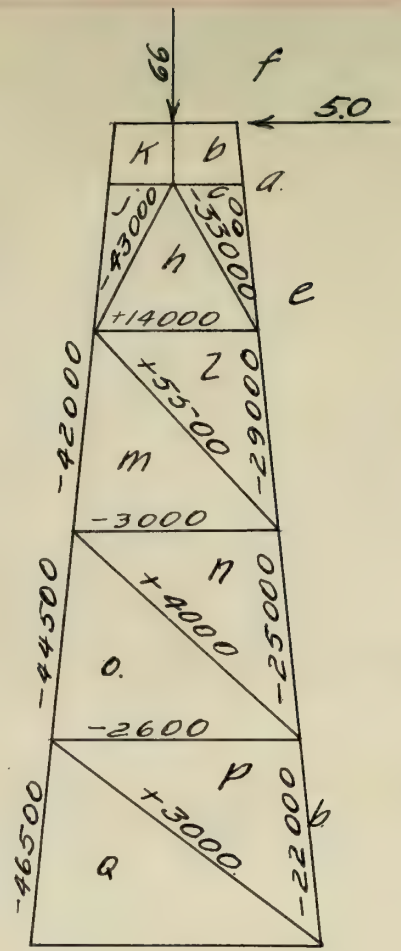
DETERMINATION
of the
LIVE LOAD CONCENTRATION.





STRESS DIAGRAM.
for.
LIVE LOAD.
on
SIDE BENT.

0 1 2 3 50001bs.

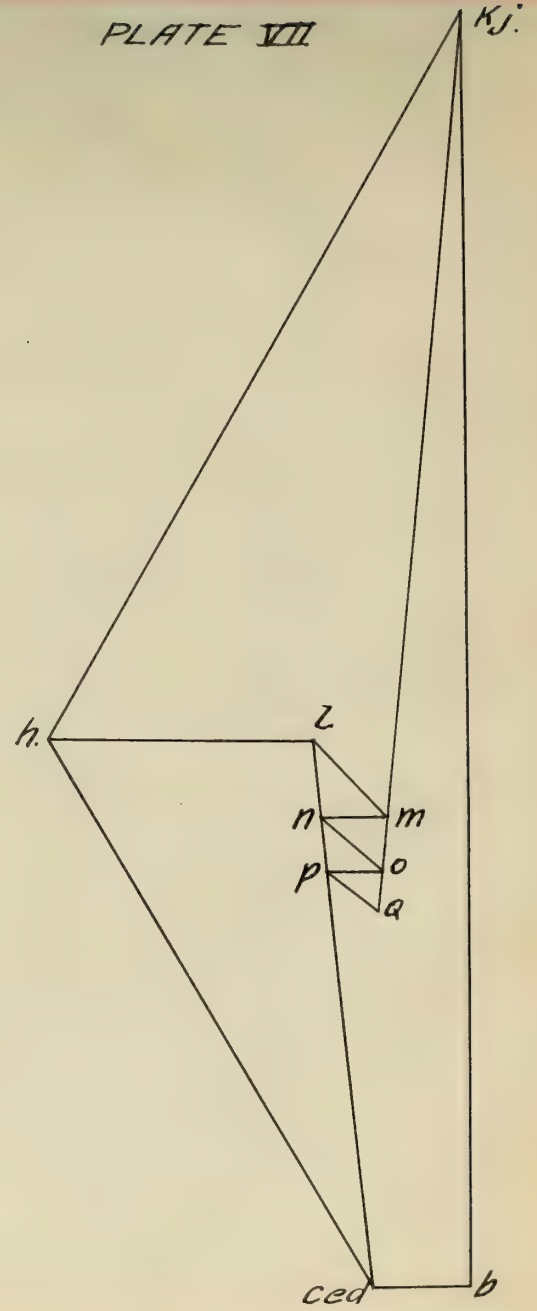


$M_a = 5 \times 5000$
$= 25000$ lb ft.

$M_b = 5000 \times 8$
$= 40000$ lb ft.

STRESS DIAGRAM
for
CENTRE VERTICAL LOAD
AND HORIZONTAL THRUST.
ON
BACK BENT.

0. 1. 2. 3. 5000 lbs.



joints are made by grooved cast-iron (See Plates II, III, IV, and V.)

B, LIVE LOAD STRESSES.

The live load is considered as acting through the axis of the sheaves at the three points of bearing; also as acting in the direction of the resultant of the maximum stress, due to the pull on the cable.

The axle of the sheaves under a bearing stress on one cable, and a crushing stress on the other would act as a continuous beam, but on account of the difficulties of the construction of movable bearings, and also to be on the safe side, it will be considered as two simple beams.

The unequal loading on the sheaves creates a horizontal thrust which causes stresses in the diagonal bracing of the rear bents.

For the determination of the live

Load stresses - see Plate No. 8.

C DEAD LOAD STRESSES.

The dead load will for the most part cover on the four main posts. The lateral bracing will be considered as having no appreciable dead load.

In determining the dead load a preliminary design will be made. This will be done by considering the average stress in the rear main posts, due to the live and wind loading.

Sections same as obtained for the rear posts, will be used for those in front. This is done in order to provide for the effect of the great oscillations, which come on a structure of this character, and also to provide for the stresses due to derail tipple, a breaker or any other structure that may in the future be connected to the head frame.

ART. 4. PRELIMINARY DESIGN.

MAIN POSTS.

The main posts will be composed of a section, consisting of four Z-bars and one web plate. This kind of a column is considered to be of such a character best suited to the conditions as, on account of its large radius of gyration per square unit of section, it can economically withstand the effect of the vibrations, and also it is very accessible for efficient painting.

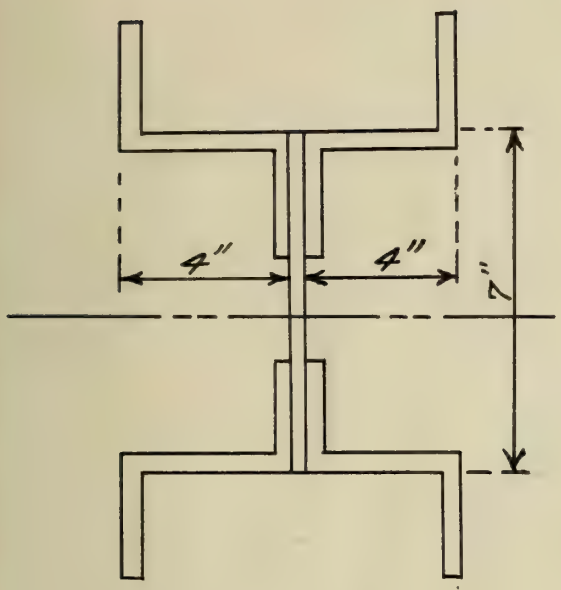
It is required to design the main post in order to determine the dead load. The average stress in the post due

to mid and the load is 110000 pounds. Try an 8 in I-bar column, composed of four L-bars $4'' \times 4'' \times \frac{3}{8}''$ and one web plate $7'' \times \frac{3}{8}''$. The length is 16 feet. The least radius of gyration, r, is 2.57 inches, the area 17.1 sq in and the weight 58 pounds per linear foot.

Taking the radius of gyration as 2.57 inches, the value of $\frac{L}{r}$ equals $16 \times \frac{12}{2.57} = 74.5$. P equals 6650 pounds per sq in, and the required sectional area, 16.5 sq in, is somewhat less than the area of the column chosen, 17.1 sq in.

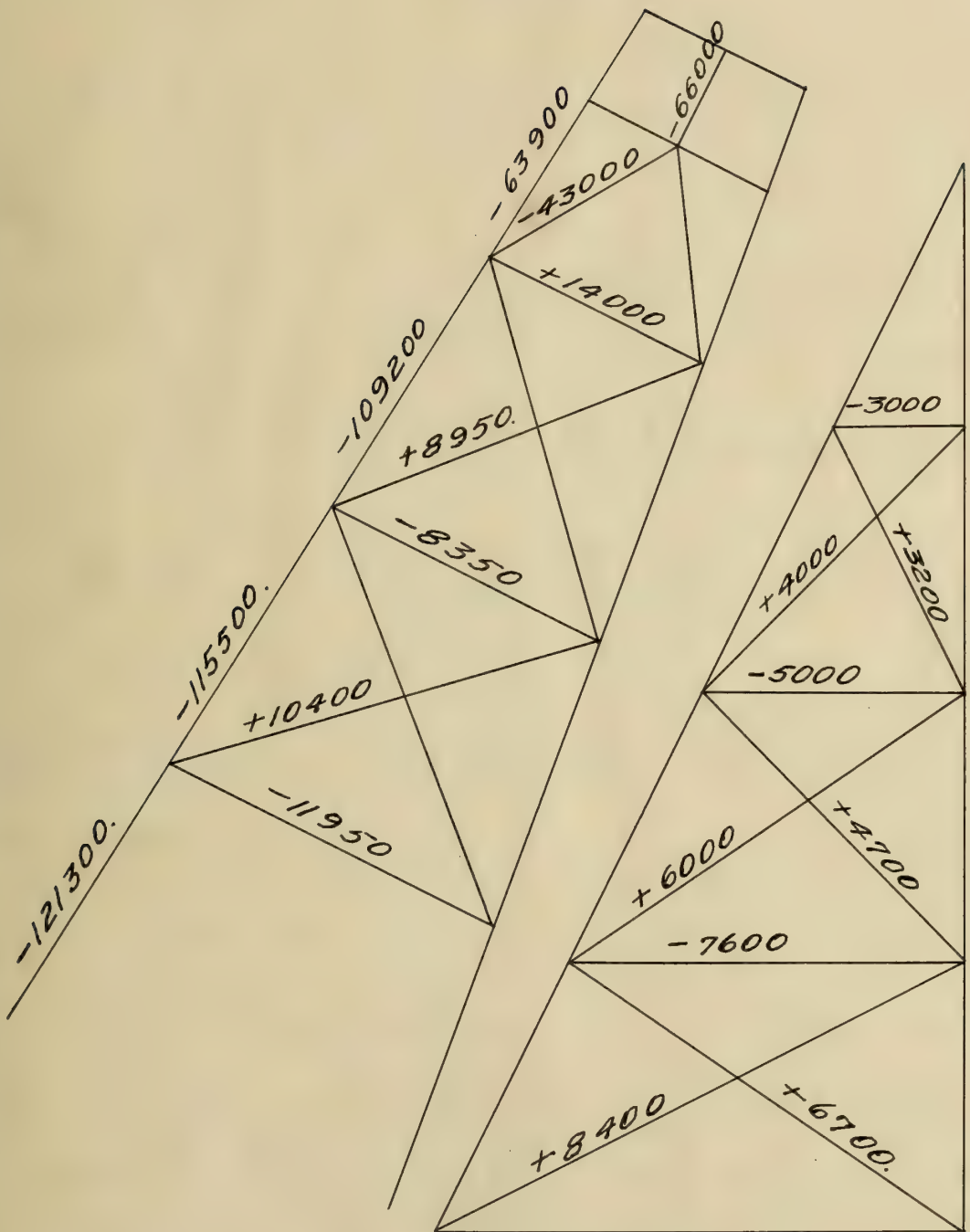
This section is sufficiently close, as great nicety is not required in preliminary design, and will be used to determine

The dead load.



CROSS-SECTION OF COLUMNS.

To allow for the weight of connections, rivets and gusset plates, 20% will be added, thus making the total weight of a column equal to $928 + 0.20 \times 928 = 1115$ pounds.



MAXIMUM STRESSES.

ART. 5. FINAL DESIGN.

A. Main Post Section.

For the three bottom sections of the main posts, the same section will be used as is required for the lower one.

The total stress to be considered is 121300 pounds, the length is 16 feet. The post will consist of a section composed of four Z-bars 4" x 7/16" and one web plate 7" x 7/16". The least radius of gyration is 2.49 inches, and the area 19.0 sq ins. The value of $\frac{l}{r}$ equals $\frac{16 \times 12}{2.49} = 77.1$ which gives a value of 6530 pounds for the allowable unit stress. The required sectional area is therefore $\frac{121300}{6530} = 18.6$ sq ins. As the assumed sectional area is

slightly larger than the required area, this section is considered as being sufficiently strong.

As previously mentioned, the front posts are considered as having the same stress as the rear posts. The section given above will be used for all of the main posts.

B - Lateral Bracing.

The lateral bracing between the rear posts, will be considered starting from the bottom and going towards the top in succession.

The first member has a stress of 11950 pounds in compression and a length of 18.9 feet. A section consisting of four $4'' \times 3'' \times \frac{3}{16}''$ angles placed in pairs with their short legs back to back, with $\frac{1}{4}$ inch lacing, will

be used. Back to back of angles is 10 inches. The least radius of gyration 1.79 inches, gives a value of 118 $\frac{L}{r}$. The allowable unit stress is 4900 pounds, and this value divided into the total stress gives a required sectional area of 2.10 sq ins.

This section will also be used for the second horizontal members.

The stress to be considered in the third horizontal member is 14000 pounds in tension. A trial shows that a 4"x4"x$\frac{3}{8}$" angle is sufficiently large, when allowing only one leg as being effective.

All other diagonal bracing being in tension, a trial shows that 4"x4"x$\frac{3}{8}$" angle is sufficiently strong in each case.

= C.-SIDE LATERAL BRACING =

The maximum stress in the horizontal members is 7600 pounds in compression; length 22 feet 5 inches. By a trial it is shown that the section

used for the horizontal members between the rear posts, is sufficient in this case.

In the diagonal members there is a maximum tension stress of 8400 pounds. A single 4"×4"×$\frac{3}{8}$" angle will be used for each member, and each pair will be riveted at their plane of intersection.

The same sections will be used for the front lateral bracing, as was used for the rear bracing.

Guides and Bracing.

The stresses in the guides and bracing are rather indeterminate. However, we know that the wind stress, and stresses due to the friction of the carriages, will be taken up by these members.

The design is such that practice and experience has shown it to be sufficient for all practical purposes.

D-SHEAVE.

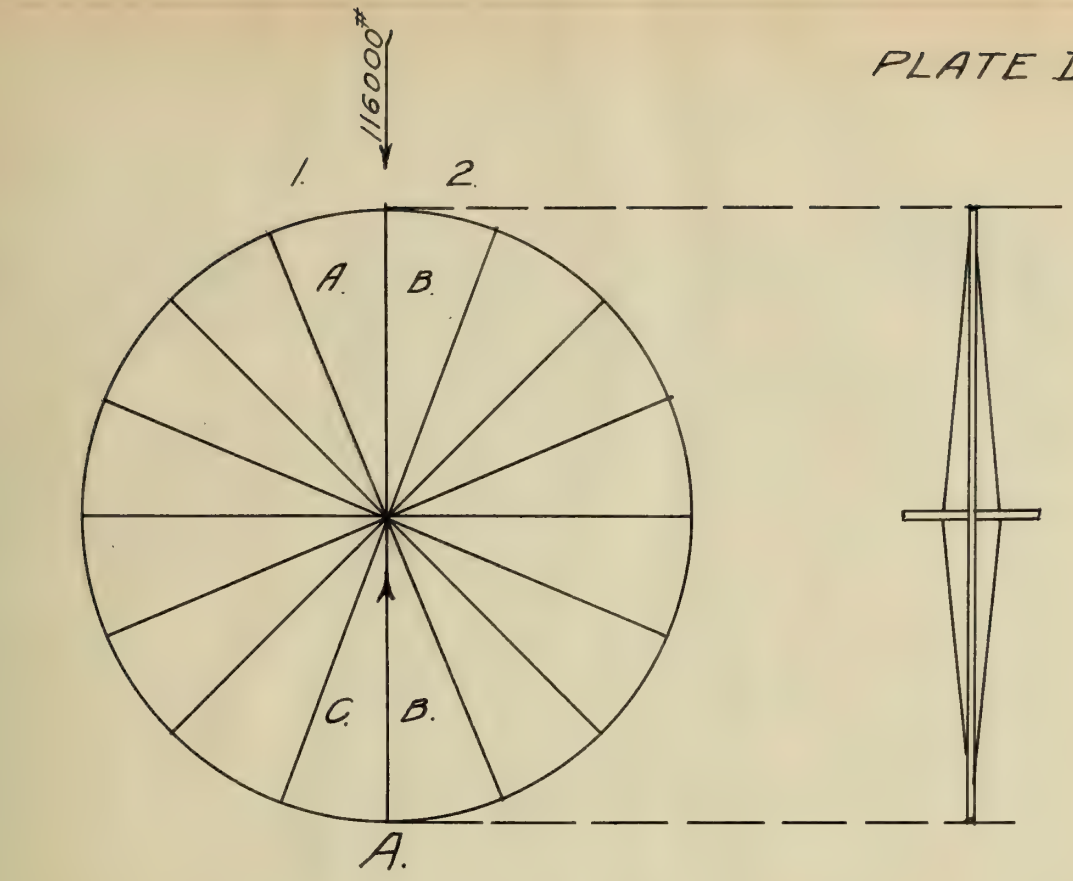
The sheave having sixteen spokes is designed similar to a bicycle wheel having tensile spokes only, as explained in Merriman's and Jacoby's "Roof and Bridges" Part III. The stress diagram on Plate IX gives a tensile stress of 55000 pounds in each spoke. The area of cross-section of a steel rod to take this stress is 1.5 sq inches. Rods 1 inch in diameter will be used. The sheave is designed as a simple beam uniformly loaded with 116000 pounds between two adjacent spokes. From $M = \frac{SI}{c}$ letting S = 8000 pounds per sq in. The area required is 10 sq inches.

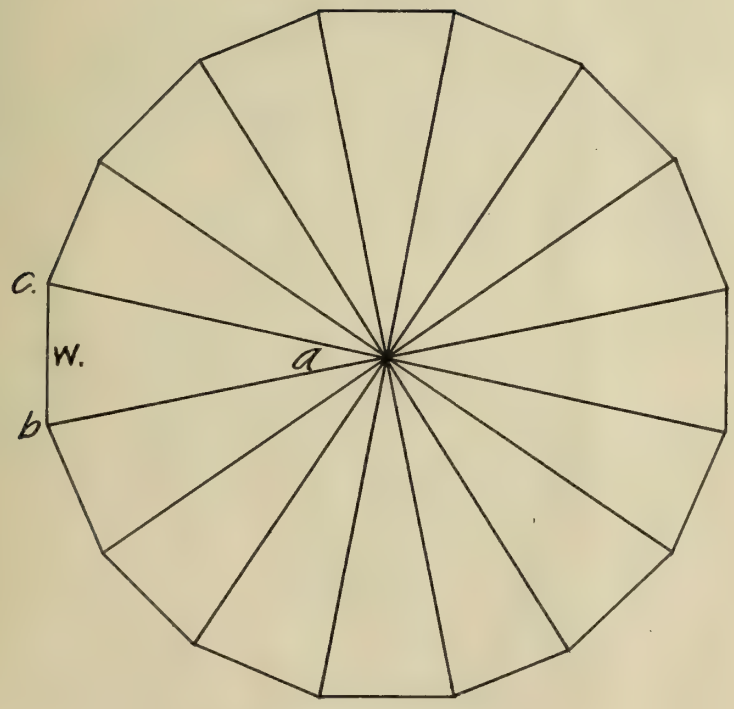
E- AXLE.

The axle is treated as a simple beam having a distance of 5 feet between supports. Using a unit working stress of 15000 pounds per sq in gives a required diameter of 5 inches.

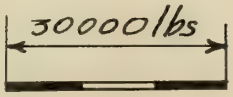
is connected to the headphones.

PLATE IX.





STRESSES
in the
SHEAVES.



ESTIMATE. OF WEIGHT.

Ref No	NAME OF PIECE.	NO	SIZE	Length in feet.	Wt in lbs per lin ft.	Weight pet piece.	
1	Z-Bar Columns	2	8"	66.5	64.7	8500	
2	" " "	2	8"	58.0	"	7500	
3	" " "	2	6"	11.0	59.9	1320	
4	" " "	2	6"	9.0	"	1080	
5	" " "	2	6"	5.0	"	600	
6	Angles	4	4"x4"x$\frac{5}{8}$"	20.0	9.8	780	
7	"	4	"	22.0	"	860	
8	"	4	"	18.0	"	700	
9	"	4	"	22.0	"	860	
10	"	4	"	28.0	"	1100	
11	"	12	"	16.0	"	630	
12	"	2	"	12.0	"	470	
13	"	8	4"x3"x$\frac{5}{16}$"	15.0	7.2	860	
14	"	8	"	18.0	"	1030	
15	"	8	"	6.5	"	380	
16	"	8	"	14.0	"	800	
17	"	8	"	21.0	"	1200	
18	Channels	2	6"	58.0	8.00	930	
19	"	7	"	8.0	"	450	
20	I. Beams.	3	"	58.0	12.25	2150	
21	Axle.	1	"	11.0	96.14	1060	
22	Sheaves.	2	@ 1600# =			3200	
							36060
			20% For connections. =			7210	
			4% For rivets. =			1440	
							8650
					Total Weight =		44710

ESTIMATE OF COST.

Cost of Materials.

44700# @ 1.84 cts — $822.60

Shop Cost.

22.3 Tons @ $15.00 — 334.50

Erection Cost.

22.3 Tons @ $13.00 — 289.90

Drafting

22.3 Tons @ $4.00 — 89.20.

Hauling.

22.3 Tons 25 mi @ 25 cts per ton mile. — 139.38

Painting.

22.3 Tons @ $1.00 per ton — 22.30

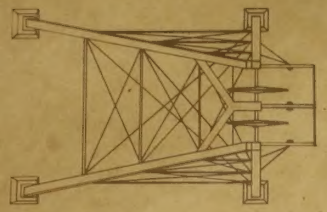
Total = $1697.86

$\frac{\$1697.86}{44710}$ = 3.8 ¢ per lb.

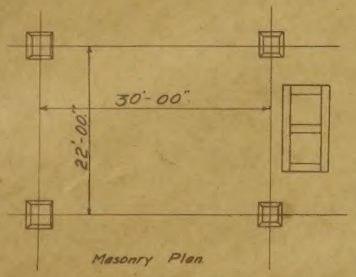
= Average pound price.

PLATE X.

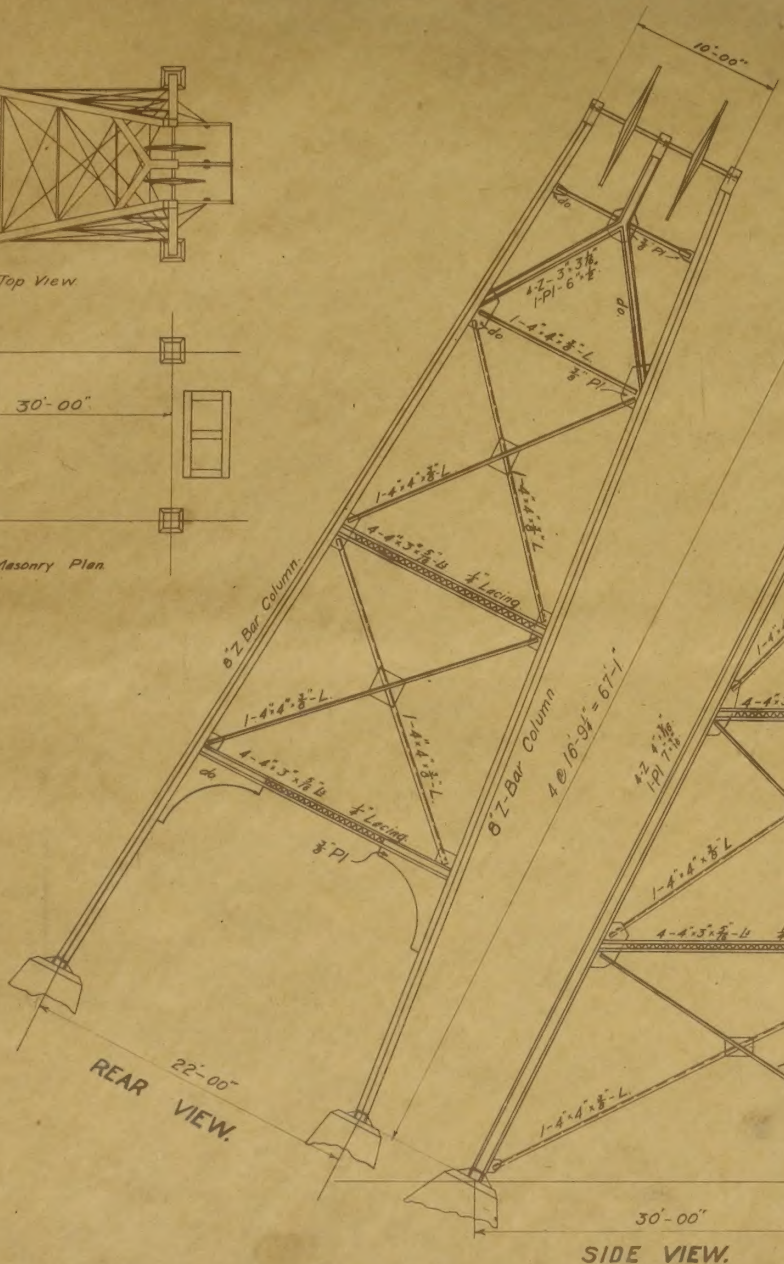
GENERAL DRAWING.

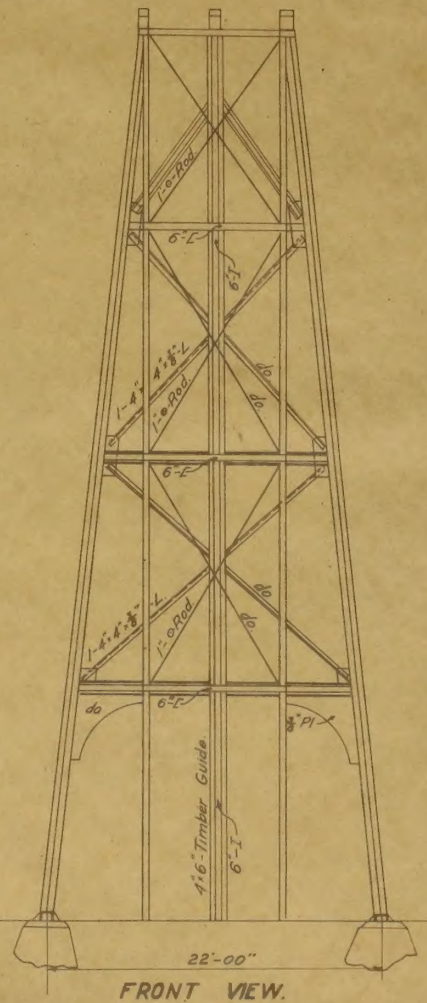


Top View.



Masonry Plan.





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